TABLE 105. CONSTRUCTION AND OPERATION AND MAINTENANCE COST FOR A 25 Mgal/d HIGH GRADIENT MAGNETIC SEPARATION INSTALLATION[a] [28]

Construction cost	
Total, $	2 113 000
$/Mgal·d	84 500
Operation and maintenance cost	
$/yr	544 000
$/1 000 gal treated	0.12

a. ENR 2000.

Mgal/d x 0.0438 = m^3/s
1 000 gal x 3.785 = m^3

BIOLOGICAL TREATMENT ALTERNATIVES

Biological treatment is a means of removing organic pollutants from wastewater streams, and can be accomplished either aerobically or anaerobically. Several biological processes have been applied to combined sewer overflow treatment, including: contact stabilization, trickling filters, rotating biological contactors (RBC), and treatment lagoons [2].

Biological systems must be operated continuously to maintain an active biomass or be able to borrow the biomass from a system which does operate continuously. This and the high initial capital costs are serious drawbacks in utilizing biological systems in stormwater treatment.

Development and testing of new biological treatment processes and further demonstration of established stormwater biological systems at other locations have not been attempted beyond the original demonstration facilities. Complete descriptions, including design criteria, process performance costs, and facilities descriptions, have previously been evaluated [2]. The following contains a summary of each process, using updated information and data, when available, of completed biological facilities.

Process Description and Installations

Descriptions of the biological processes used to control the organic pollutants found in stormwater are summarized in Table 106. These biological systems are generally located adjacent to conventional biological facilities for a source of biomass, with the possible exception of treatment lagoons. Contact stabilization, trickling filters, and RBCs require supplemental treatment, usually final clarification, to remove the biological solids generated by the process. Effluent from treatment lagoons may also require additional treatment for control of algae or floatable solids. Descriptions of typical biological treatment installations are summarized in Table 107.

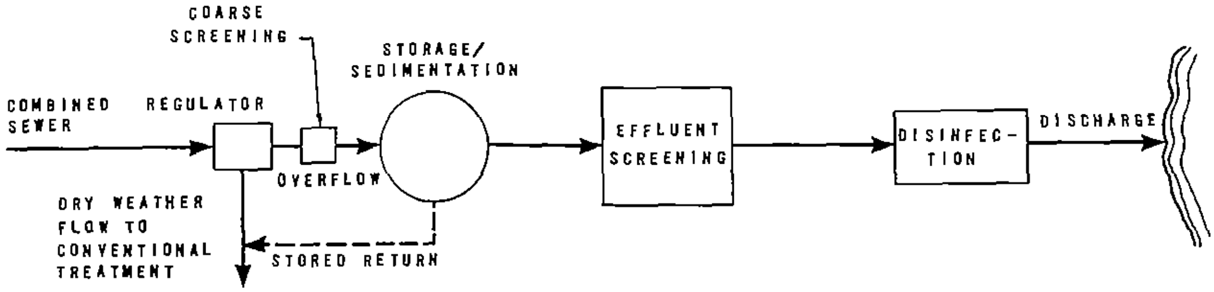


Figure 52. Typical process flow diagram for sedimentation.

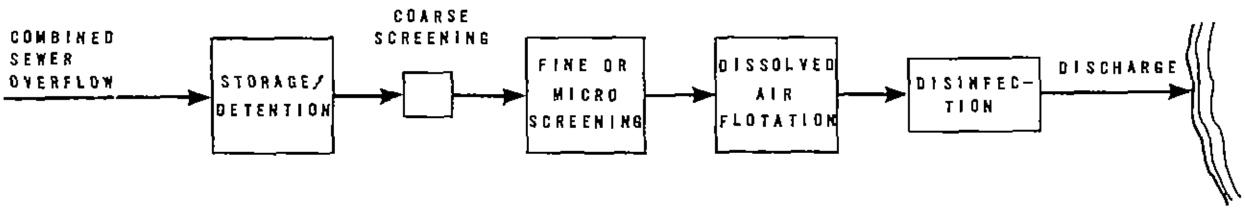


Figure 53. Typical process flow diagram for dissolved air flotation.

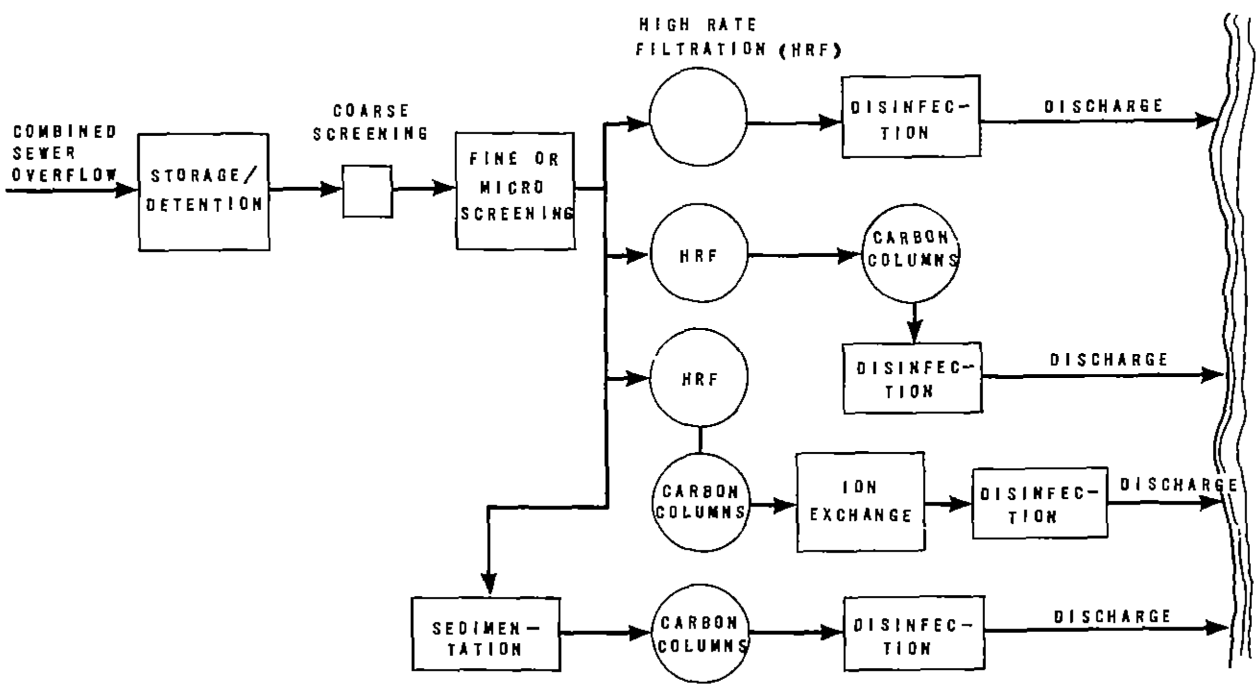


Figure 54. Typical process flow diagram for several advanced physical/chemical treatment systems.

TABLE 106. DESCRIPTION OF BIOLOGICAL PROCESSES USED IN COMBINED SEWER OVERFLOW TREATMENT [2, 64]

Biological process	Process description	Source of biomass	Requires additional treatment	Type of additional treatment
Contact stabilization	Process is a modified activated sludge process in which the absorption phase, or contact, and the oxidation phase (stabilization) takes place in two separate tanks. Sludge is wasted from the stabilization tank to maintain constant biomass concentrations.	From conventional activated sludge treatment facility.	Yes	Secondary clarification
Trickling filters	Standard trickling filter process in which a biological growth is supported on a stationary medium and the stormwater distributed over the surface and allowed to flow through the media. Process can include standard rate or deep bed plastic media designs.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Rotating biological contactors	Process operates on the same principle as trickling filters; however, the biological growth is supported on large diameter, closely spaced disks which are partially submerged and rotate at slow speeds.	Must be continuously maintained with a source of food.	Yes	Secondary clarification
Treatment lagoons				
Oxidation ponds	Shallow aerobic ponds which rely on surface reaeration for oxygen supply to maintain biological uptake of organics. Sedimentation also occurs in oxidation ponds.	Allowed to generate for each storm.	Usually	Algae removal by screening or filtration
Aerated lagoons	Similar to oxidation ponds except they are deeper and rely on artificial means of oxygen supply such as surface aerators or diffused air systems. System operates under aerobic conditions.	Allowed to generate for each storm.	Yes	Final clarification or sand filtration
Facultative lagoon	Facultative lagoons are the deepest of the lagoons and rely on surface reaeration. The lagoons have three distinct layers: aerobic near the surface due to algae and reaeration, a transition zone, and an anaerobic zone near the bottom sludge deposits. The biological oxidation and anaerobic stabilization occur simultaneously.	Allowed to generate for each storm.	Yes	Final clarification, screening, or sand filtration

TABLE 107. SUMMARY OF TYPICAL BIOLOGICAL STORMWATER TREATMENT INSTALLATIONS

Project location	Type of biological treatment	Tributary area, acres	Design capacity, Mgal/d	Major process components	No. of units	Total size	Period of operation
Kenosha, Wisconsin [72]	Contact stabilization	1 200	20	Contact tank	2	32 700 ft^3	1972 to 1975
				Stabilization tank	2	97 900 ft^3	
Milwaukee, Wisconsin [71]	Rotating biological contactors	35	0.05[a]	3 ft diameter RBC units	24	28 300 ft^2	1969 to 1970
Mt Clemens, Michigan [52]							
Demonstration system	Treatment lagoons in series with recirculation between storms	212	1.0[b]	Storage/aerated lagoon	1	750 000 ft^3	1972 to 1975
				Oxidation lagoon	1	1 100 000 ft^3	
				Aerated lagoon	1	930 000 ft^3	
Citywide full-scale system	Storage/treatment lagoons in series with recirculation between storms	1 471	4.0[b]	Aerated storage basin	1	4 440 000 ft^3	Under construction
				Aerated lagoon	1	508 000 ft^3	
				Oxidation lagoon	1	1 100 000 ft^3	
				Aerated/oxidation lagoon	1	922 000 ft^3	
New Providence, New Jersey [73]	Trickling filters	.	6.0	High-rate plastic media	1	36 ft diameter	1970 to present
				High-rate rock media	1	65 ft diameter	
Shelbyville, Illinois [74]	Treatment lagoons:						
	Southeast site	44	28[c]	Oxidation lagoon	1	255 600 ft^3	1969 to present
	Southwest site	450	110	Detention lagoon plus 2-cell facultative lagoon	1	2 782 700 ft^3	1969 to present
Springfield, Illinois [75]	Treatment lagoon	2 208	67	Storage/oxidation lagoon	1	5 330 000 ft^3	1969 to present

a. Design based on average dry-weather flow, average wet-weather flow - 1 Mgal/d

b. Design flowrate through lagoon systems Total flowrate to facilities is 64 Mgal/d for the demonstration project and 260 Mgal/d for citywide system.

c. Estimated using a 50% runoff coefficient at a rainfall rate of 1.95 in./h

acres x 0 405 = ha
Mgal/d x 0.0438 = m^3/s
ft^3 x 0 0283 = m^3
ft^2 x 0 0929 = m^2
ft x 0.305 = m
in /h x 2 54 = cm/h

Evaluation of Biological Treatment Processes

Biological treatment processes are generally categorized as secondary treatment processes, capable of removing between 70 and 95% of the BOD_5 and suspended solids from waste flows at dry-weather design flowrates and loadings. When biological treatment processes are used for stormwater treatment, removal efficiencies are lower and are controlled to a large degree by hydraulic and organic loading rates. Most biological systems are extremely susceptible to overloading conditions and shock loads as compared to physical treatment processes. However, rotating biological contactors have achieved high removals at flows 8 to 10 times dry-weather design flows [71].

Biological Treatment Performance--

Typical pollutant removals for contact stabilization, trickling filters, and RBCs are presented in Table 108, for wet-weather loading conditions. These processes include primary and final clarification. Final clarification greatly influences the overall performance of the system by preventing the carryover of biological solids produced by the processes.

TABLE 108. TYPICAL WET-WEATHER BOD AND SUSPENDED SOLIDS REMOVALS FOR BIOLOGICAL TREATMENT PROCESSES

	Expected range of pollutant removal, %	
Biological treatment process	BOD	Suspended solids
Contact stabilization	70-90	75-95
Trickling filters	65-85	65-85
Rotating biological contactors[a]	40-80	40-80

a. Removal reflects flow ranges from 30 to 10 times dry-weather flow.

Average pollutant removal by the contact stabilization process at Kenosha, Wisconsin, is presented in Table 109. Pollutant removal effectiveness was shown to be directly dependent on the quality of the sludge being produced by the dry-weather treatment facilities. Dry-weather activated sludge is wasted to the stabilization tank to provide the biological solids when the contact stabilization system begins operation. Only after the demonstration system has operated for many hours will the sludge in the stabilization tank actually be that produced by the demonstration system and be acclimated to the waste characteristics of wet-weather flows. The dry-weather treatment plant efficiency was also improved by utilization of the demonstration project final clarifier during periods when the demonstration facilities were not in use. Dry-weather plant efficiencies increased from 82 to 94% for BOD, and from 64 to 88% for suspended solids [72].

The plastic media and conventional rock media trickling filters at New Providence, New Jersey, operate in series during dry weather, and are operated in parallel during wet weather [73]. When the system is operated in the parallel mode, overall average pollutant removal is decreased and is affected by the hydraulic flow to the plant, as shown in Figure 55. Overall pollutant removal also includes both primary and final clarification. It was also demonstrated that the plastic media filter removed about 2.7 times the BOD as compared to the rock media filter during wet-weather flows: approximately 0.86 kg BOD/m^3 as compared to 0.32 kg BOD/m^3 (54 lb/1000 ft^3 versus 20 lb/1000 ft^3) at a 45% BOD removal efficiency. A comparison of the BOD removal efficiency as a function of hydraulic and organic loading rates for the rock media and the plastic media trickling filters is shown in Figure 56.

TABLE 109. AVERAGE POLLUTANT REMOVAL PERFORMED FOR THE KENOSHA, WISCONSIN, CONTACT STABILIZATION FACILITY [72]

	Influent[a]	Effluent[b]	Removal %
Suspended solids, mg/L	299	23	90.4
Suspended volatile solids, mg/L	148	13	90.0
Total solids, mg/L	685	464	29.2
Total volatile solids, mg/L	252	130	41.6
Total BOD, mg/L	119	16	84.8
Dissolved BOD, mg/L	31	7	72.1
COD	366	66	81.9
Total organic carbon, mg/L	117	23	76.5
Dissolved organic carbon, mg/L	29	15	39.7
Kjeldahl nitrogen as N, mg/L	13.70	7 6	43.7
Total phosphate as P, mg/L	4.64	1.8	58.6
Total coliforms, MPN/mL	31 038	3 726	
Fecal coliforms, MPN/mL	2 238	443	

Note: All values indicated are arithmetic mean of 30 runs at acceptable operating levels except for coliforms which are geometric means.

a. Influent samples taken from grit tank effluent.

b. Effluent samples taken prior to chlorination

The demonstration scale RBC at Milwaukee, Wisconsin, confirmed pilot plant results, handling a higher range of organic and hydraulic loads for periods of 8 to 10 hours [71]. A comparison of organic removal efficiency for both the pilot plant studies (using raw sewage) and the full-scale wet-weather demonstration facilities is shown in Figure 57. It was also shown that as hydraulic residence times fell below about 8 to 10 minutes, the organic removal efficiency of the demonstration facility dropped significantly. This treatment system was installed as an inline device without final clarification. Final clarifiers could greatly increase BOD and suspended solids removal by removing the sloughed biological mass caused by the high hydraulic loadings.

Lagoon Treatment Performance--

Pollutant removal efficiencies by treatment lagoons have varied from highs of 85 to 95% to negative values due to excessive algae production and carryover. In addition to the type of lagoon and the number of cells in series (stages), several major factors that influence removal efficiencies include: (1) detention time, (2) source of oxygen supply, (3) mixing, (4) organic and hydraulic loading rates, and (5) algae removal mechanisms [2, 52, 74, 75].

A single cell storage/oxidation lagoon in Springfield, Illinois, averaged 27% BOD removal and 20% suspended solids removal; however, fish kills in the receiving water were greatly reduced as compared to that prior to the construction of the facility [75]. Multiple cell facilities with algae control systems constructed at Mount Clemens, Michigan and Shelbyville, Illinois provide 75 to 90% suspended solids and BOD removal efficiencies during wet-weather conditions [52, 74].

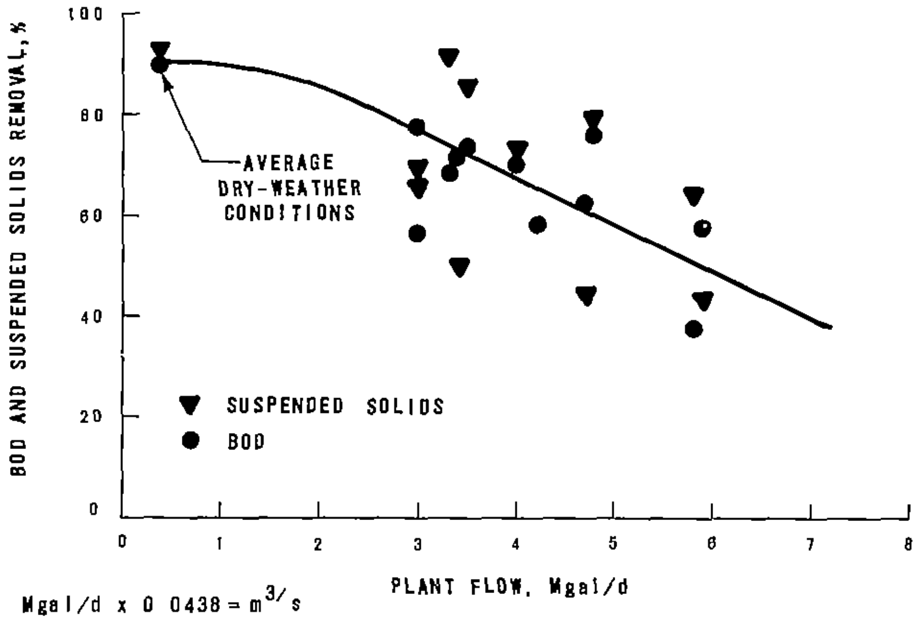


Figure 55. Overall trickling filter performance as a function of hydraulic flow, New Providence, New Jersey [73].

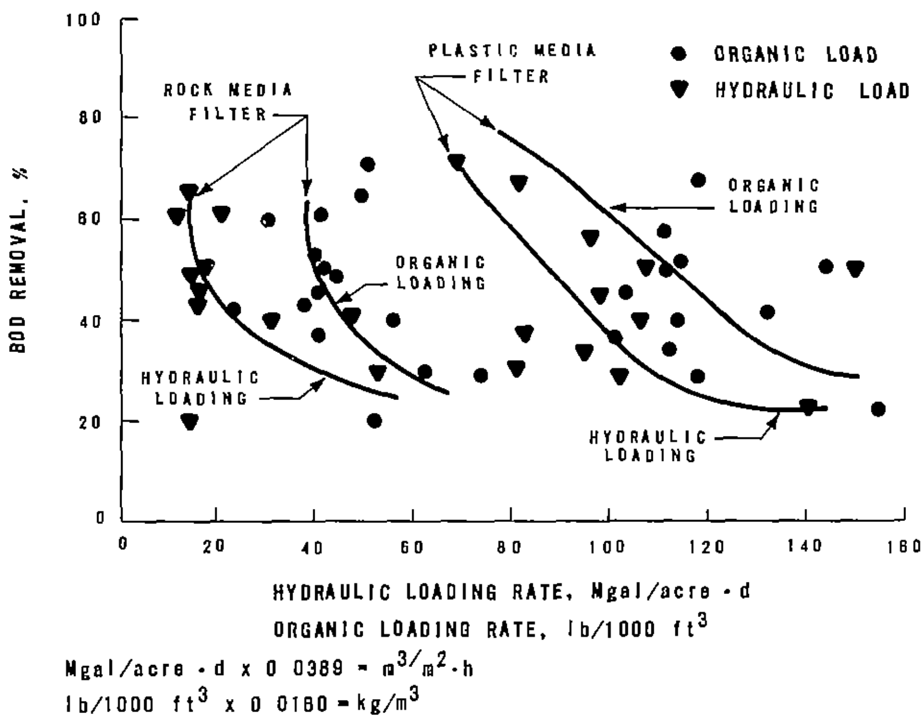
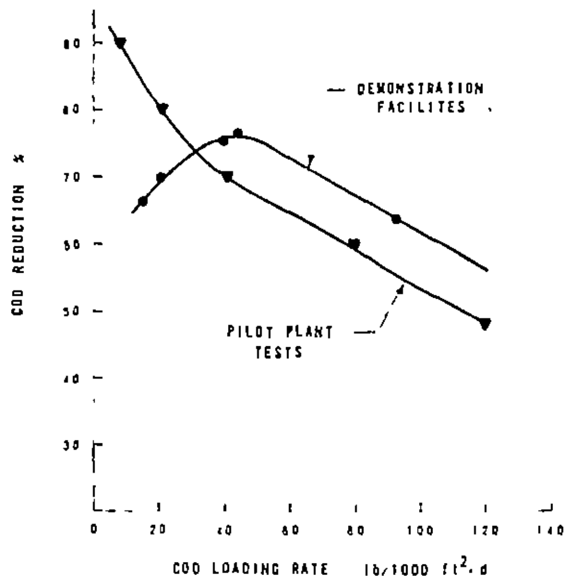


Figure 56. Comparison of rock media and plastic media trickling filters as a function of hydraulic and organic loading rates [73].



lb/1000 ft^2·d x 4.882 x 10^{-3} = kg m^2·d

Figure 57. Comparison of COD removal performance for pilot and full scale demonstration RBC facilities, Milwaukee, Wisconsin [71].

Process efficiency profiles for suspended solids and BOD at the Mount Clemens demonstration facility are shown in Figure 58, for a 3-stage lagoon system with a microstrainer and sand filtration for suspended solids, BOD, and algae control. It was determined that intermediate algae control had little effect on the overall treatment performance [52].

Operational Problems--

An operational problem common to all stormwater biological systems is that of maintaining a viable biomass to treat flows during wet-weather conditions. For processes that borrow biomass from dry-weather facilities or allow the biomass to develop, a lag in process efficiency may be experienced as the biomass becomes acclimated to the changing waste strength and flowrate. In addition to maintaining a biological medium, clarification and/or storage are often required to provide operational control of the process, and can greatly increase capital costs of the facility.

General maintenance problems experienced by wet-weather biological facilities are similar to those experienced at conventional biological installations. Winter operation of mechanical surface aerators have had some serious drawbacks, including icing, tipping, or sinking [52, 72]. Other methods of providing the required oxygen that show promise and have been demonstrated at many dry-weather facilities include diffused air systems and submerged tube aerators [2].

At Mount Clemens, Michigan, operational problems included sludge buildup in the first cell of the lagoon system and algae control [52].

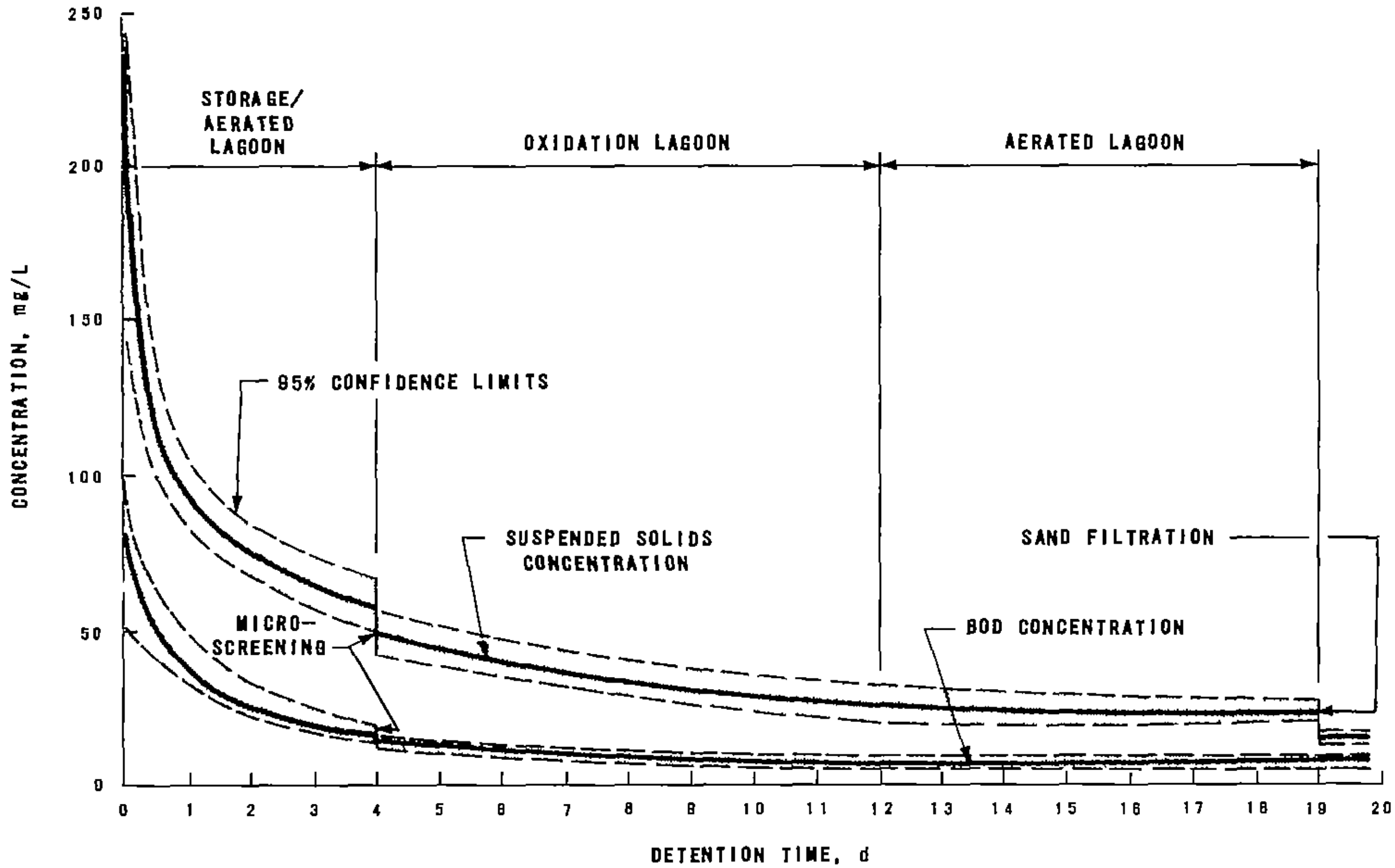


Figure 58. Suspended solids and BOD_5 concentration profiles for a 3-stage lagoon treatment system, Mount Clemens, Michigan [52].

Design Criteria--

The principal design criteria used to evaluate and design biological systems generally include hydraulic and organic loading rates, sludge and hydraulic detention times, and in the case of contact stabilization, such factors as F/M ratio, mass of organisms in the system, and rate of substrate utilization.

At Kenosha, Wisconsin, several process criteria were correlated with effluent BOD and suspended solids concentrations and removal efficiencies. The results of this correlation are presented in Table 110 [72]. These tests also indicated that low MLSS concentration of less than 2100 mg/L and high reaeration times of greater than 4 hours and long stabilization periods may seriously affect process efficiency. A contact time of at least 10 minutes was also found for satisfactory operation and performance of the facilities.

TABLE 110. RESULTS OF CORRELATION OF CONTACT STABILIZATION PROCESS PERFORMANCE AND PROCESS PARAMETERS AT KENOSHA, WISCONSIN [72]

Process equation		Multiple correlation coefficient
Effluent BOD concentration, mg/L	= 1.6 (A) + 0.92 (B) + 9.1	0.670
Effluent SS concentration, mg/L	= 2.43 (C) + 1.83 (A) + 13.9	0.544
BOD removal, %	= 0.081 (D) - 1.0 (B) - 1.3 (A) + 80.6	0.745
SS removal, %	= 0.02 (E) - 0.97 (C) - 0.7 (A) + 87.1	0.691

Note: A = F/M ratio
B = Stabilization time, d
C = Reaeration time, h
D = Influent BOD concentration, mg/L
E = Influent SS concentration, mg/L

Typical design criteria for biological treatment systems have been previously presented and discussed in the literature [2] and are summarized in Tables 111 through 114. Design criteria for treatment lagoons are not based on biological kinetic theory, but rather on actual practice and experience. An inventory and operational data from municipal lagoon facilities have been collected for various types of lagoons for each region in the United States [76]. Factors affecting lagoon performance, including organic and hydraulic loading, odor and aesthetic failures, wind, light, and mixing, are evaluated.

TABLE 111. OPERATIONAL AND DESIGN PARAMETERS FOR THE CONTACT STABILIZATION FACILITY AT KENOSHA, WISCONSIN [72]

Parameter	Average value[a]	Range of values tested[b]
MLSS concentration, mg/L	3 400	1 000-5 600
F/M ratio in contact tank, lb BOD_5/lb MLSS·d	2 8	0.5-5.0
Sludge retention time, d	2.3	0-7.0
BOD loading rate, lb BOD_5/1 000 ft^3·d	500	200-1 000
Detention time, h		
Contact tank	0.25	0.17-0.33
Reaeration time	3.0	1.0-10.0
Recycle ratio, Qr/Q	0.40	0.20-0.60
Volume of air supplied in contact tank, ft^3/lb BOD_5	250	100-700

a. Based on 30 optimized runs.

b Ranges based on 49 runs.

lb BOD_5/lb MLSS d = kg BOD_5/kg MLSS·d
lb BOD_5/1 000 ft^3·d x 0 016 = kg BOD_5/m^3·d
ft^3/lb BOD_5 x 62.4 = L/kg BOD_5

TABLE 112. DESIGN CRITERIA FOR TRICKLING FILTERS OPERATED IN PARALLEL FOR CONTROL OF WET-WEATHER FLOWS [2, 73]

Parameter	High rate	Ultra-high rate
Filter media	Rock	Plastic[a]
Hydraulic loading rate, Mgal/acre d		
Pecommended design	20	70
Range	10-40	40-120
Organic loading rate, lb BOD_5/1 000 ft^3		
Recommended design	40	85
Range	20-115	45-230
Depth, ft	3-8	20-40[b]
Recirculation ratio, Qr.Q	1:1-4·1	1:1-4:1

a. Or redwood slats

b Ultra-high rate trickling filter depth at New Providence, New Jersey = 14.4 ft.

Mgal/acre·d x 0.039 = m^3/m^2 h
lb BOD_5/1 000 ft^3 d x 0.016 = kg BOD_5/m^3·d
ft x 0.305 = m

TABLE 113. COMPARISON OF DRY-WEATHER AND WET-WEATHER DESIGN PARAMETERS FOR ROTATING BIOLOGICAL CONTACTORS

Parameter	Range of general dry-weather values [2]	Milwaukee, Wisconsin [71]	
		Dry-weather design	Wet-weather range
Hydraulic loading rate, gal/ft^2·d[a]	2-8	1 8	25-35
Organic loading rate, lb BOD_5/1 000 ft^2·d	5-15	5.4[b]	30-70[b]
Detention time, min	15-20	69	10-20

a. Based on disc surface area

b. Based on correlation of COD:BOD ratios.

gal/ft^2·d x 1.698 = L/m^2·h
lb BOD_5/1000 ft^2·d x 4.882 x 10^{-3} = kg BOD_5/m^2·d

TABLE 114. COMPARISON OF DESIGN CRITERIA FOR TREATMENT LAGOONS [2]

	Oxidation lagoons	Aerated lagoons		Facultative lagoons
		Aerated oxidation lagoon	Complete mix aerated lagoon	
Organic loading rate, lb BOD_5/acre·d	20-50	100-500	500-1 000	15-80
No. of lagoons	2-6	2-6	1-4	2-10
Depth, ft	2-5	6-10	10-15	6-12
Detention time, d	30-160	5-11	1-8	7-120[a]

a. Use of mechanical surface aerators reduces detentions to approximately 7-10 days.

lb BOD_5/acre d x 1.1208 = kg BOD_5/ha·d
ft x 0.305 = m

Costs of Biological Treatment Facilities--

A comparison of construction, and operation and maintenance costs for biological treatment systems and treatment lagoons is presented in Table 115. Costs of final clarification are included where control of solids and sludge produced by the biological treatment system are required. Costs also include pumping, disinfection, and algae control systems when applicable. Engineering, administration, and land costs are not included in the estimates; however, land costs may be the controlling economic factor in the evaluation of lagoon treatment systems and therefore must be evaluated for each specific locations.

Many biological treatment systems are integrated with or are a part of dry-weather treatment facilities. Cost estimates of the wet-weather portion of these facilities were separated from total costs of the total treatment

TABLE 115. SUMMARY OF CAPITAL AND OPERATION AND MAINTENANCE COSTS FOR BIOLOGICAL TREATMENT ALTERNATIVES[a]

Project location	Type of biological treatment	Peak plant capacity, Mgal/d	Construction cost, $	Cost/capacity, $/Mgal·d	Cost/tributary area, $/acre	Annual operation and maintenance cost, ¢/1 000 gal (except as noted)
Kenosha, Wisconsin [72]	Contact stabilization	20	1 364 000	68 200	1 140	13 8
Milwaukee, Wisconsin [2, 71][b]	Rotating biological contactor	4.3	299 000	69 200	8 540	4 4
Mount Clemens, Michigan [52]						
Demonstration system	Aerated treatment lagoons	64	642 700	10 000	3 030	20 0
Citywide system	Storage/aerated treatment lagoons	260	5 737 000	22 000	3 900	19.0
New Providence, New Jersey [2, 73][c]	High-rate trickling filter	6	475 000	79 150	. ..	12 3
Shelbyville, Illinois [2, 74]						
Southeast site	Oxidation lagoon	28	43 400	1 550	1 000	$1 530/yr[d]
Southwest site	Storage and facultative lagoons	110	337 700	3 070	750	$5 780/yr[d]
Springfield, Illinois [2, 75]	Oxidation lagoon	67	176 000	2 600	80	$2 100/yr

a. ENR 2000.

b Includes estimate of final clarifier.

c Includes plastic media trickling filter, final clarifier, plus one-half of other costs.

d Based on estimated man-day labor requirements.

Mgal/d x 0 0438 = m^3/s
acres x 0.405 = ha
¢/1 000 gal x 0 264 = ¢/m^3

systems. The cost of the inline RBC at Milwaukee, Wisconsin, was used together with an estimated cost for a final clarifier to develop an estimated cost of a complete RBC treatment system [71]. The final clarifier cost was based on one 19.8 m (65 ft) diameter clarifier with a surface loading rate of 2.04 $m^3/m^2 \cdot h$ (1200 $gal/ft^2 \cdot d$).

Costs of lagoon treatment systems vary widely, and are a function of the type of lagoon (oxidation, aerated, or facultative); the number of cells; and the miscellaneous equipment requirements including: aeration equipment, disinfection equipment, instrumentation, pumping, and algae control provisions.

Costs for many of these stormwater facilities are based on only one installation of each biological treatment process. Therefore, these costs should be considered only coarse estimates and may be greatly influenced by the degree of integration with dry-weather treatment required to produce a viable system. These costs can be used as a preliminary guide, but detailed analysis should be performed to compare and evaluate biological treatment alternatives with other methods of treatment and control.

Biological Treatment Systems

Both single purpose and dual use (integrated biological treatment) facilities have been demonstrated in controlling combined sewer overflows. Single purpose facilities treat flows only during wet-weather conditions as in the case of the contact stabilization installation and several lagoon installations [52, 72, 75]. However, the clarifier of the contact stabilization facilities is also used for dry weather final clarification [72]. Dual use or integrated facilities are capable of treating both dry- and wet-weather flows.

Dual use has been accomplished by changing modes of operation during wet weather as demonstrated at New Providence, New Jersey. Increased performance during dry-weather was also obtained by using the trickling filters in series [73]. Biological systems have also been used to treat dry- and wet-weather flows without process modification by pushing the system to design limits as hydraulic and pollutant loads increase. Examples include the inline RBC unit at Milwaukee, Wisconsin, and the Southwest lagoon treatment system at Shelbyville, Illinois [71, 74]. At Ft. Wayne, Indiana, an existing terminal lagoon is used by both the dry-weather treatment facilities and the wet-weather screening installation prior to discharge to the receiving water [50].

Because of the limited ability of biological systems to handle fluctuating and high hydraulic shock loads, storage/detention facilities preceding the biological processes may be required. Storage/detention will be used at the citywide lagoon treatment facilities under construction at Mount Clemens, Michigan [52]. The storage unit will reduce the maximum flows entering the system from 11.39 m^3/s (260 Mgal/d) to a design flowrate of approximately 0.18 m^3/s (4.0 Mgal/d) through the lagoon system. A similar concept is also used at the Southwest treatment site in Shelbyville, Illinois [74].

Initial capital investments of integrated or dual use facilities can be reduced by apportioning part of the costs to the dry-weather facility. The cost reduction is in proportion to the net benefit that the wet-weather facility provides to the overall treatment efficiency during dry-weather periods. A description of this evaluation is presented in Section 4.

LAND TREATMENT OF STORMWATER

Land treatment methods have been used successfully to treat municipal and some kinds of industrial wastes for several years. The use of land treatment in treating wastewater or stormwater is usually limited by hydraulic application rates and the resulting land area requirements. Since stormwater volumes can be many times larger than dry-weather municipal wastewater flows, application rates are proportionally more critical in determining the economic feasibility of their application to stormwater treatment. Unless adequate flow equalization could be provided, slow rate land treatment processes with low application rates would require excessive land area.

Process Description and Facilities Installations

Based on the limitations of application rates and land area only, the following land application processes appear to have promise for treating stormwater runoff:

- Wetlands
- Rapid infiltration
- Overland flow

These methods should have application for stormwater treatment despite the absence of conclusive design, operating, and performance data from operational projects.

Wetlands--

Wetlands are areas with too many plants and too little water to be called lakes, yet they have enough water to prevent most agricultural or silvicultural uses. Existing wetlands areas are generally large enough to accommodate expected stormwater runoff volumes and their ability to influence stormwater quality appears to hold promise.

The Wayzata, Minnesota [77], project is one of the few projects currently investigating the potential of wetlands treatment, but any conclusions regarding expected quality will require more data. However, results from wetlands projects researching the potential for renovating municipal wastewater indicate effective treatment does take place [78]. The management technique for nutrient removal, loading rates, and the suitable site characteristics need further study. Winter application in northern latitudes may not be feasible.

Rapid Infiltration--

In rapid infiltration, most of the applied wastewater percolates through the soil, eventually reaching the groundwater. Rapidly permeable soils such as sands and loamy sand are suited to this process. The high application rates preclude consumptive use by plants (vegetative covers are not normally used) and there is little evaporation. Return of renovated water to the surface by wells, underdrains, or groundwater interception may be necessary or may be an advantage depending on existing groundwater quality reuse potential or water rights considerations. Rapid infiltration is only affected by the most severe climatic conditions and will require a relatively small amount of land if soil conditions are correct. Surface clogging due to high suspended solids loading can reduce infiltration rates and may require pretreatment.

Overland Flow--

In overland flow treatment, water flows across a vegetative surface to runoff collection ditches for reuse or discharge to surface water. Treatment is by physical, chemical, and biological means as a thin film of water flows over the relatively impermeable surface; very little percolation takes place.

Land Application Projects--

The only actual stormwater land treatment projects discovered in the literature are a pilot scale wetlands treatment system in Wayzata, Minnesota [77], and an experimental scale project in Tucson, Arizona [79], which combined the rapid infiltration and overland flow methods. Features of these projects are shown in Table 116.

TABLE 116. DESCRIPTION OF STORMWATER TREATMENT PROJECTS USING LAND TREATMENT

Item	Wayzata, Minnesota [77]	Tucson, Arizona [79]
Type of treatment	Wetlands	Overland flow, rapid infiltration
Hydraulic loading, Mgal/acre·yr	$\sim$2.4[a]	140-880
Land area, acres	7.5	0.02
Period of operation	November 1974 to present	Four trials, fall 1971
Preapplication treatment	Gravel roughing filter	None
Vegetative cover	Marsh vegetation	Turf grass
Surface influent and effluent monitoring	Yes	Yes
Groundwater monitoring	Observation wells and lysimeter pans	Subsurface flow collected by underdrain for monitoring
Management techniques	Intermittent application, dewatering, recirculation, and comparison with unmanaged control marsh	Four separate trials monitored changes in surface and subsurface outflow quality with respect to time

a. Hydraulic loading includes surface runoff (1.12), precipitation (0.83), and groundwater infiltration (0.42).

Mgal/acre·yr x 9 353 6 = m^3/ha·yr
acre x 0 405 = ha

These projects indicate that significant renovation is taking place, but more data are needed to support any conclusions on expected quality of the treated stormwater, pretreatment requirements, marsh fill-in, vegetation maintenance and control, and associated costs.

Evaluation of Land Treatment Alternatives

Although limited data have been compiled, an evaluation of the various land treatment alternatives using available data from stormwater treatment projects and municipal dry-weather flow projects is presented for pollutant removal efficiencies, design criteria, and costs.

Process Evaluation--

Results comparing treatment of domestic wastewater by natural and artificial marshes indicated that significant pollutant removals take place in each case [80]. It was determined that artificial marshes acted similar to natural marshes, but treatment efficiency was better for managed artificial systems. Removals were related to detention time and the length of marsh through which the wastewater passed. Treatment efficiency was adversely affected by climatic conditions; poor pollutant removals associated with the first heavy frost of the fall were observed. The best seasonal removals averaged approximately 29% for BOD and 13% for phosphorus for natural marshes. The managed artificial marsh averaged approximately 90% for BOD and 64% for phosphorus. Marsh systems can handle the high solids loading associated with stormwater runoff, and management techniques to increase pollutant removals are available.

Studies using marsh systems for stormwater treatment also indicate significant pollutant removals, as summarized in Table 117.

TABLE 117. TYPICAL POLLUTANT LOADING AND REMOVAL RESULTS USING LAND TREATMENT

	Wayzata, Minnesota [77]		
	Pollutant loading, lb/acre·yr	Removal, %	Tucson, Arizona [79]
Suspended solids	4,973	94	Results indicated significant pollutant removal, but loading and percent removals were not determined.
Phosphorus	17.8	78	
Ammonia-nitrogen[a]	64.8	0	

a Ammonia concentrations in groundwater are higher than the stormwater influent.

lb/acre·yr x 1.121 = kg/ha·yr

Limited studies using stormwater runoff and rapid infiltration indicate good treatment performance, however, actual percent removals were not determined [79]. Several conclusions can be made from results using sanitary wastewaters:

- Pollutant removals by the filtering and straining action of the soil are excellent.
- Suspended solids, BOD, and fecal coliforms are almost completely removed.
- Nitrogen removals are generally poor unless specific operating procedures are established to maximize denitrification.

Total nitrogen removals range from 30%, without denitrification procedures, to 50% if steps to maximize denitrification are taken. Phosphorus removals can range from 70 to 90% depending on the physical and chemical characteristics of the soil.

Overland flow systems can achieve treatment to secondary level (or better) from raw, primary and treated, or lagoon treated municipal wastewater. Nitrogen and BOD removals are comparable to conventional advanced wastewater treatment. Nitrogen removals usually range from 75 to 90% with runoff nitrogen being mostly in the nitrate form. Nitrogen removal can be affected by cold weather as a result of decreased plant uptake and reduced biological activity. Phosphorus removals by adsorption and precipitation are limited because of incomplete contact between the wastewater and the adsorption sites within the soil; removals usually range from 30 to 60% on a concentration basis.

Design Criteria--

Applying alternative land treatment methods to stormwater treatment will be affected to different degrees by climatic restrictions, constituent and hydraulic loading to the system (i.e., preapplication treatment), site characteristics, and vegetative cover. Typical design features for the various processes, based on treatment of municipal wastewater, are compared in Table 118. The major site characteristics are compared for each land treatment process in Table 119.

The nitrogen, phosphorus, suspended solids, and BOD loading capacity will vary for each land treatment process depending on such factors as preapplication treatment, expected treatment performance, hydraulic limitation of the soil and underlying geology, nitrogen removal capacity of the soil-vegetation complex, and discharge standards.

For rapid infiltration systems, the infiltration capacity of the soil could be limited by excessive suspended solids loadings. If rapid infiltration is used, it is recommended that stormwater suspended solids concentrations be consistent with that of primary treated municipal effluent before application to the land. Nitrogen loading is often the limiting criterion for percolating water from rapid infiltration systems to meet EPA drinking water standards of 10 mg/L for nitrate-nitrogen. Crop uptake of nitrogen, denitrification, and storage in the soil will all affect the maximum allowable loading. Other loading parameters may include phosphorus and heavy metals.

For overland flow systems, treatment performance is directly related to pollutant loadings and hydraulic application rates. The general pollutant loading capacity depends primarily on the expected treatment performance and the level of preapplication treatment. Suspended solids reductions to a level consistent with municipal wastewater that has been screened and possibly degritted and degreased would be desirable to ensure successful operation of the system. Methods for distribution of stormwater runoff with high suspended solids loads will require careful consideration. Because application rates partially govern the expected effluent quality, maximum allowable application rates during precipitation may be relatively low. As a result, significant storage may be required affecting the economic feasibility of this process.

TABLE 118. COMPARISON OF DESIGN FEATURES FOR LAND TREATMENT PROCESSES [78]

Feature	Application process		
	Wetlands	Rapid infiltration	Overland flow
Application techniques	Sprinkler or surface	Usually surface	Sprinkler or surface
Annual application rate, ft/yr	4 to 100	20 to 560	10 to 70
Field area required, acres[a]	11 to 280	2 to 56	16 to 110
Typical weekly application rate, in./wk	1 to 25	4 to 120	2.5 to 16
Minimum preapplication treatment provided in United States	Primary treatment or coarse filtration	Primary sedimentation	Screening and grit removal
Disposition of applied wastewater	Evapotranspiration, percolation, and runoff	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required

a. Field area in acres not including buffer area, roads, or ditches for a 1 Mgal/d (43.8 L/s) flow.

ft/yr x 0.3048 = m/yr
acres x 0.405 = ha
in./wk x 2.54 = cm/wk

TABLE 119. COMPARISON OF SITE CHARACTERISTICS FOR LAND TREATMENT PROCESSES [78]

Characteristics	Application process		
	Wetlands	Rapid infiltration	Overland flow
Slope	Usually less than 5%	Not critical, excessive slopes require much earthwork	Finish slopes 2 to 8%
Soil permeability	Slow to moderate	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)
Depth to groundwater	Not critical (zero)	10 ft (lesser depths are acceptable where underdrainage is provided)	Not critical
Climatic restrictions	Storage may be needed for cold weather	None (possibly modify operation in cold weather)	Storage often needed for cold weather

ft x 0.3048 = m

Costs of Land Treatment Systems--

There is an absence of full scale operational projects where capital and operating costs have been compiled. However, cost curves for rapid infiltration and overland flow systems which treat municipal wastewater have been compiled presenting component capital and operating costs [81].

The use of existing wetlands already influenced by stormwater would appear to be very economical but existing sites are not always available. Creation of artificial wetlands is another approach which has received some attention as a low cost land treatment method.

DISINFECTION

Disinfection of storm and combined sewer overflows is generally practiced at all stormwater treatment facilities to control pathogens and other microorganisms in receiving waters. At most stormwater installations, disinfection has been accomplished by applying conventional wastewater technology supplemented by high rate processes and on-site generation of disinfectant. Several aspects of disinfection practices require reconsideration for stormwater treatment applications. These include:

- A residual disinfecting capability may not be feasible for stormwater discharges. Recent work indicates that chlorine residuals and compounds discharged to natural waters may be harmful to aquatic life.

- The coliform count is increased by surface runoff in quantities unrelated to pathogenic organism concentration. Total coliform levels may not be the most useful indication of disinfection requirements and efficiencies.

- Discharge points requiring disinfection are often at outlying points on the sewer system and require unmanned, automated installations.

- Storm flow is highly variable both in quantity and quality; disinfection facilities must be able to meet these fluctuations.

Three basic needs for control of microorganisms in stormwater overflows have been identified [82]: (1) to obtain knowledge of the storm flow's microorganism pathogenic quality and the pathogens' relationships to other indicator organisms; (2) to develop high-rate disinfection systems to reduce large tankage and/or dosage requirements, and (3) to develop disinfection facility design and operation techniques for the highly varying quality and quantity characteristics of storm flows.

Disinfection Projects

Demonstration projects evaluating stormwater disinfection technology are summarized in Table 120. Other projects, evaluating the characteristics and impacts of microorganisms in stormwater, have been beneficial in providing a background understanding of the sources and constituents of microbial contamination in overflows [82-85].

TABLE 120. SUMMARY OF DEMONSTRATION STORMWATER DISINFECTION PROJECTS

Project location	Disinfectant agent	Source	Description of disinfection system	Period of operation
Boston, Massachusetts [17]				
Cottage Farm Detention and Chlorination Station	Sodium hypochlorite (NaOCl)	Purchased/stored	Automatic disinfection system injects up to 3 000 gal of 10 to 15% NaOCl into the influent channel to the detention basins for the design storm.	1971 to present
Cleveland, Ohio [86]	Sodium hypochlorite (NaOCl)	Purchased/stored	Disinfection of two bathing beaches enclosed by fabric barriers and disinfection of polluted streams and overflow points influent to Lake Erie.	1968 to 1970
Fitchburg, Massachusetts [87]	Sodium hypochlorite (NaOCl)	Purchased/stored	High-rate application of disinfectant via thin film in a Dynactor. System incorporates chemically assisted high-rate settling.	1974 to present
New Orleans, Louisiana [88]	Sodium hypochlorite (NaOCl)	Central generation	NaOCl is generated at a central manufacturing facility with a capacity of 1 000 gal/h. The 12% NaOCl is transported and stored at 4 pumping stations on 3 overflow channels to disinfect pumped stormwater.	1972 to present
New York City, New York [25]				
Spring Creek	Sodium hypochlorite (NaOCl)	Purchased/stored	Automatic disinfection system injects up to 60 000 lb/d of 5% NaOCl into the inlet sewer of the storage/detention facilities.	1972 to present
Philadelphia, Pennsylvania [55, 56, 57]	Sodium hypochlorite (NaOCl) Ozone (O_3)	Purchased On-site generation	Comparison of two disinfectants on screened and unscreened combined sewer overflow. Short contact times are achieved by high velocity gradients in a plug flow contact chamber regime.	1969 to 1973
Rochester, New York [36]	Chlorine (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Sequential addition of Cl_2 and ClO_2 with flash mixing at each point of application. Disinfection is final treatment step following sedimentation, storage, dual media filtration, and carbon column pilot facilities.	1975 to 1976
Syracuse, New York [35, 89]	Chlorine gas (Cl_2) Chlorine dioxide (ClO_2)	Purchased On-site generation	Evaluation of individual and sequential addition of Cl_2 and ClO_2 following treatment of combined sewer overflows by screening and swirl concentration.	1974 to present

gal x 3.785 = L
lb/d x 0.454 = kg

The Fitchburg, Massachusetts, demonstration facility represents a new technology in disinfectant application [87, 90]. The 373 m^3/d (100 000 gal/d) combined sewer treatment facility includes chemical addition ($FeCl_3$, CaO, and polymer) and high-rate settling prior to disinfection.

Disinfection is accomplished by the use of thin film technology. Hypochlorite is sprayed on a thin film of wastewater to provide maximum instantaneous contact and eliminate the need for further mixing. A small sump is provided at the outlet of the unit but no contact chamber is required. Analysis indicates that both total coliform and fecal coliform are reduced to less than 36 organisms per 100 mL.

A second high-rate settling unit after disinfection was found to add little to the overall suspended solids removal efficiency. Typical pollutant removals for the facility average 65% for BOD and COD, 85% for suspended solids, 90% for total phosphorus, and over 99% for total and fecal coliforms.

Future studies proposed at Fitchburg will include the use of ozone as a disinfecting agent.

Disinfection Agents

The disinfection agents used in wastewater and stormwater treatment include chlorine, calcium and sodium hypochlorite, chlorine dioxide, and ozone. Results from combined bench and pilot plant testing of high gradient magnetic separators indicate 99.9% removal of viruses and over 99% removal of total and fecal coliforms [28]. However, physical methods and other chemical agents have not experienced wide usage either because of excessive costs or difficulties with application technology.

Evaluation of Disinfection Agents--

The four potential disinfection agents have some common characteristics; all are oxidizing agents, corrosive to equipment, and are highly toxic to both microorganisms and higher life. Other characteristics and differences that should be considered when choosing a stormwater disinfectant are summarized in Table 121. A discussion of these characteristics follows.

Stability--The more stable chemicals allow the designer greater flexibility in developing a treatment facility. Chlorine gas is always purchased and its high degree of stability allows long storage periods. Hypochlorite can be purchased or generated onsite and can be stored for several months, or it can be generated at a steady rate and stored between overflow events. Peak demand requirements can come from storage or be purchased as needed.

At New York's Spring Creek facility, purchased sodium hypochlorite is diluted and stored at a strength of about 5% available chlorine, which reduces the rate of deterioration [25]. It has been shown that the stability of sodium hypochlorite is higher at reduced concentrations [2]. Chlorine dioxide and ozone are the least flexible; they must be generated onsite and their effective lives are too short to make storage practical. Consequently, disinfectant generating capacity must be sufficient to handle anticipated peak demands.

TABLE 121. CHARACTERISTICS OF PRINCIPAL STORMWATER DISINFECTION AGENTS

Characteristic	Chlorine	Hypochlorite	Chlorine dioxide	Ozone
Stability	Stable	6 month half-life	Unstable	Unstable
Reacts with ammonia to form chloramines	Yes	Yes	No	No
Destroys phenols	At high concentrations	At high concentrations	Yes	Yes
Produces a residual	Yes	Yes	Short lived[a]	No
Affected by pH	More effective at pH < 7.5	More effective at pH < 7.5	Slightly	No
Hazards	Toxic	Slight	Toxic, explosive	Toxic

a. *Chlorine dioxide dissociates rapidly [89].*

Chemical Reactions--Chlorine and hypochlorite will react with ammonia to form chloramines and with phenols to form chlorophenols. These are toxic to aquatic life and the latter also produces taste and odor in the water. Chlorine dioxide does not react with ammonia and completely oxidizes phenols. Ozone is also more effective in oxidizing phenols. Elimination of possible odor and taste problems will be important for water reuse or recyle.

Residuals--Chlorine and hypochlorite disinfection have a residual that can be easily measured, will provide protection against aftergrowth, and can be used to control application rates without biological testing. Chlorine dioxide dissociates rapidly and can be measured by election spin resonance (esr), a procedure that requires more elaborate equipment than measuring common chlorine residuals and by modified DPD techniques [89]. Ozone does not have a measurable residual and would have to be controlled by biological testing.

pH Effect--Both chlorine and hypochlorite disinfection depend on the HOCl phase of the chlorine-water reaction. This phase predominates at low pH and is essentially absent above a pH of 9; therefore, these agents are limited to acidic or neutral wastewaters. A study at Fort George Meade, Maryland, showed that both coliform and virus removal was improved with pH adjustment to the acid range [91]. The effective pH range for chlorine dioxide lies between 4.5 and 7.5 [89]. It should be noted, however, that adjustment of pH below 5 may be unacceptable to prevailing stream conditions and standards, and may not be economically justifiable. Ozone is relatively unaffected by pH and can be used on any waste or at any point in the treatment scheme.

Hazards--Chlorine, chlorine dioxide, and ozone are all dangerous gases that must be carefully handled by competent personnel. The hazards of chlorine gas are well known and have caused restrictions of its use or transport in several cities including New York and Chicago. Gas concentrations as low as 5 ppm can cause difficulty in breathing and 1000 ppm can be toxic. Chlorine dioxide has toxicities similar to chlorine gas and the additional danger of exploding with any slight change in environment. It must be kept in the aqueous state to minimize dangers. The gas is soluble in water but does not react chemically with water. Ozone's oxidizing capacity makes concentrations of 1.0 ppm in the atmosphere hazardous to health. Hypochlorite can be obtained as a solid or liquid and does not have the potential dangers of the other three agents. It is the safest choice for remote, unmanned disinfection operations.

Evaluation of Application Technology--

Several studies have been conducted to examine application techniques that improve or enhance the disinfecting capability. Adequate mixing under plug flow conditions and sequential addition of chlorine (Cl_2) and chlorine dioxide (ClO_2) were two significant parameters which influenced disinfection efficiency.

Mixing--In high-rate disinfection systems where contact times are less than 10 minutes, usually in the range of 1 to 5 minutes, adequate mixing is a critical parameter, providing complete dispersion of the disinfectant and forcing disinfectant contact with the maximum number of microorganisms. The more physical collisions high-intensity mixing causes, the lower the contact time requirements. Mixing can be accomplished by mechanical flash mixers at the point of disinfectant addition and at intermittent points, or by specially designed contact chambers, or both [2, 36, 55].

At Philadelphia [55, 57], a specially designed contact chamber with closely spaced corregated baffles was used to increase the velocity gradient (G) in t^{-1}. G is a function of the viscosity of the fluid, velocity, and headloss. In this application it was considered desirable to keep the product of G and detention time (t) a constant, at less than peak design flow conditions. Assuming that t remains constant, therefore velocity remains constant, G is increased by increasing the headloss through the use of corregated channels [2]. Spacing and arrangement of the channels is also essential to maintain plug flow conditions preventing any backmixing of the dispersed disinfectant. Using this design, a contact time of 3 minutes with initial chlorine concentrations as low as 2.6 mg/L, reductions of total and fecal coliforms by 99.9% were obtained.

At an experiment at Fort George Meade to show the effect of mixing on disinfection, turbulence was created in a sewage effluent line by installing a 20.3 cm (8 in.) orifice to increase flow velocities to the range of 2 to 2.3 m/s (6.6 to 7.6 ft/s). Virus kills were increased to 83.6 to 99.3% from 45.8 to 73.5%; however, it was found that coliform kills did not substantially increase [91].

Sequential Addition of Disinfectants--Disinfection was shown to be enhanced beyond the expected additive effect by sequential addition of Cl_2 followed by

ClO_2 at intervals of 15 to 30 seconds [36, 82, 89]. A minimum effective combination of 8 mg/L of Cl_2 followed by 2 mg/L of ClO_2 was found as effective as adding 25 mg/L Cl_2 or 12 mg/L ClO_2 individually in reducing total and fecal coliforms, fecal streptococci, and viruses to acceptable target levels [82,89].

It was surmised that the presence of free Cl_2 in solution with chlorite ions (ClO_2^-), (the oxidized state of ClO_2), may cause the reduction of ClO_2^- back to its original state. This process would prolong the existence of ClO_2, the more potent disinfectant [82, 89].

Other significant findings of the Onondaga County, New York, studies include the following:

- Sequential doses of the same disinfectant do not increase disinfection over a single dose with the same total quanity.
- Prescreening does not appear to affect Cl_2 disinfection but slightly improves disinfection with ClO_2.
- Cl_2 and ClO_2 demands may be due to different materials in wastewater.
- The maximum antiviral activity of ClO_2 was found to occur between pH 4.5 and 7.5.
- Increases of temperature from 2°C to 30°C (36° to 86°F) slightly improved high-rate bacterial disinfection with both Cl_2 and ClO_2. Viral inactivation with ClO_2 was sharply decreased at 4°C (39°F) but unaffected between 12°C and 36°C (54°F and 97°F).

Aftergrowth of Microorganisms

Aftergrowth of indicator microorganisms in stormwater after disinfection have been reported [84, 86, 89]. Indicator microorganisms, specifically total coliforms, enter a log growth phase when the disinfectant residual decreases to undetectable values. Aftergrowth coliform levels can exceed before disinfection background levels. Total and fecal coliform aftergrowth were reported during stream and laboratory studies at Cleveland, Ohio [86]. Only total coliform aftergrowth was reported during a stormwater disinfection study at The Woodlands, Texas [84]. In both cases, aftergrowth of fecal streptococci did not occur. Laboratory aftergrowth studies in Syracuse, New York, revealed that difficulties in simulating the conditions for aftergrowth may be encountered for bench scale tests [89]. Aftergrowth tests, conducted to determine the ultimate bacterial and viral counts that might result in the receiving water from the discharge of untreated and disinfected combined sewer overflow, showed no measurable increases during and up to 3 days. These results were felt to be more indicative of the inability to simulate receiving water conditions in the laboratory rather than a lack of aftergrowth.

A possible chemical change in the composition of the stormwater caused by chlorine may enhance aftergrowth. This chemical change is assumed to be a cleavage of large protein molecules into smaller proteins, peptides, and amino acids. These smaller molecules are more readily available to the bacteria for growth and reproduction than the larger proteins [86].

The City of Cleveland conducted a research study to determine the cause of the aftergrowth that occurred during the hypochlorination of the streams [86]. Also, possible methods to reduce aftergrowth were investigated. The study consisted of: (1) a stream study of bacterial aftergrowth resulting from hypochlorination, and (2) bench scale studies on possible relationships between aftergrowth and chlorination due to chlorination-induced changes. Conclusions of the bacterial aftergrowth study are summarized as follows:

- Hypochlorination of streams results in a significant reduction of indicator bacteria; however, as soon as the chlorine residual dissipates, a bacterial aftergrowth occurs.

- Fecal streptococcus exhibited a very limited ability for aftergrowth in the laboratory. Fecal coliforms displayed a moderate ability for aftergrowth. Total coliforms were capable of aftergrowth that closely approximated, or exceeded, their respective initial levels.

- Factors found to significantly affect bacterial aftergrowth are: (1) the extent of dilution of the chlorinated water; (2) time available for aftergrowth between chlorination and dilution; and (3) levels of residual chlorine.

- While maintaining a 6 mg/L chlorine residual throughout a laboratory study, no significant decrease in aftergrowth was noted by increasing the chlorination detention time from 15 minutes to 72 hours.

- Proteins, as analyzed by the Lowry Method for protein determination, were greatly increased in stream water samples upon the addition of sodium hypochlorite. It is assumed that chlorine cleaves larger protein molecules into smaller proteins, peptides, and amino acids which yield more reactive sites to react with the Lowry color development reagent. All the reactive nitrogenous organic compounds were calculated as protein. Since both laboratory and field studies show bacterial populations were greater after chlorination than before, it is further hypothesized that the smaller nitrogenous compounds were more easily utilized by the bacteria for growth and reproduction which could be significant in the rate and magnitude of bacterial aftergrowth.

- Other than of proteinaceous material, there were no appreciable chlorination induced chemical-physical changes in the water samples studied that could be demonstratably related to bacterial aftergrowth.

A multipurpose investigation of surface water quality and disinfection was conducted in a 8100 ha (20 000 acre) test site at The Woodlands, Texas [84]. It was found that following disinfection of stormwater with either chlorine.

ozone, or bromine with dosing up to 32 mg/L, aftergrowth occurred after 4 to 8 days. Aftergrowth occurred only in the total coliform group.

Biological Indicator Organisms

Total coliforms, fecal coliforms, and fecal streptococci are the most common biological indicator organisms used to measure water, wastewater, and stormwater pathogenic quality and disinfection efficiency. Because extremely high coliform counts can come from natural background sources other than humans, the use of the coliform group as an indicator of the presence of pathogens in stormwater has been questioned [84, 85]. Analysis of soil samples taken from areas adjacent to established stream sampling stations and from other areas of The Woodlands, Texas, yielded positive values for all indicator bacteria groups, including pathogens [84]. In Baltimore, investigations have also revealed little or no correlations between indicator and pathogenic bacteria in storm and stream samples; however, pathogens were received in all stormwater samples [85].

In using coliform counts to measure or control disinfection efficiency, and as a basis of design when the possibility of aftergrowth of coliform organisms exists and/or potentially high background levels exist, gross over or under design of disinfection facilities may result.

Studies have been conducted to evaluate alternative microbial indicators including high chlorine resistant organisms, pathogens themselves, fecal coliforms to fecal streptoccus ratios, and adenosine triphosphate [83, 84, 89, 92].

The coliform group of indicator organisms have a relatively low chlorine resistance when compared to such pathogens as enteric viruses and protozoan cysts. Three indicators were investigated which were resistant to chlorination in the range considered necessary for the inactivation of pathogens and viruses. These included a yeast and two acid-fast bacillus [83]. Similar studies were conducted in Syracuse, New York, using bacteriophage f2 and ØX174, Polio-1, and other viruses that are more resistant to chlorination than the coliform indicator bacteria [89].

Measurement of pathogens themselves is a method to identify microbial quality directly [85, 92]. However, procedures to isolate and enumerate viruses such as Salmonella, Shigella, Pseudomonas aeruginosa, and Staphlococcus aureus are considerably more difficult than for the coliform group. Better methods and reliable qualitative recovery procedures for the enumeration of pathogenic microorganisms should be developed to identify pathogen presence and impact in storm runoff and combined sewer overflows [85].

Measurements of fecal streptococcus in addition to total and fecal coliforms may provide an indication of the source of the polluting bacteria groups through the use of the fecal coliforms/fecal streptococci ratio (FC/FS) and fecal coliform/total coliform ratio (FC/FT) [82, 84, 85]. An FC/FT ratio of greater than 0.1 is believed to be indicative of sewage; however, a firm FC/FT ratio has been difficult to establish. An FC/FS ratio of 4.0 or greater is believed to be indicative of human sources and a ratio of 1.0 or less is believed to be indicative of animal sources. The FC/FS ratios between 0.7 and

4.0 are difficult to interpret. It is suggested that FC/FS ratios be applied carefully and that the ratios are most meaningful when data are collected at discharge points to the receiving water. Upon entering receiving waters, the levels of each of the microorganisms may be affected by numerous environmental factors and differential microbial die-away [85].

In samples of storm runoff, FC/FS ratios of less than 1.0 have been noticed and FC/FS ratios representative of combined sewer flows had only 18% of samples greater than 4.0, indicating animal sources of contamination [85].

A potential alternative to microbial indicators is the use of adenosine triphosphate (ATP), a substance that is universally found in all living cells. Significant decreases in ATP that parallel bacteria reductions have been observed during the disinfection process. It may be feasible to use ATP as an instantaneous measure and a control for disinfection processes [89].

Costs of Stormwater Disinfection Systems

Costs of disinfection systems used to treat combined sewer overflows and stormwater discharges can vary greatly depending on the complexity of the system. Stormwater disinfection must be flexible and capable of automatic operation to handle intermittent and varying flows and volumes. Summaries of typical disinfection costs are presented in the literature for chlorine gas, hypochlorite, and ozone systems [2, 27].

Costs used for disinfection alternative selection should be evaluated using local conditions and requirements. These can include disinfection and receiving water requirements amd standards, equipment and disinfectant availability and costs, and system control and operation requirements.

Improvements and changes in on-site generation equipment may make these alternatives more economically attractice for storm flow applications. Ozone generation, although more expensive than other methods of disinfection, may become an economically feasible alternative in light of increasingly strict control of residuals and compounds formed by chlorine disinfection and the increasing costs of chlorine [82].

Cost curves comparing chlorine gas, chlorine dioxide, and hypochlorite generation disinfection systems have been developed and are presented in Figure 59. These costs (ENR 2000) include manufactured equipment, piping, housing, electrical and instrumentation, and miscellaneous items. No allowance for contingency or land was included. Operation and maintenance cost curves have also been developed and include annual labor requirements; miscellaneous supply costs for chlorine gas, chlorine dioxide, and hypochlorite disinfection systems; and power requirements for hypochlorite generation [27].

ILLUSTRATIVE PROBLEMS

Comparison of several stormwater treatment technologies together with examples of process design and cost evaluations are presented in Example Problems 7-1 through 7-5. The problems include a cost-effectiveness comparison of total storage and storage/sedimentation; design of a swirl concentrator, including geometry modifications; development of an equation for estimating operation and maintenance costs for storage facilities; and a method for optimizing integrated storage/treatment facilities. An evaluation of land requirements and design considerations for land treatment of stormwater is also presented.

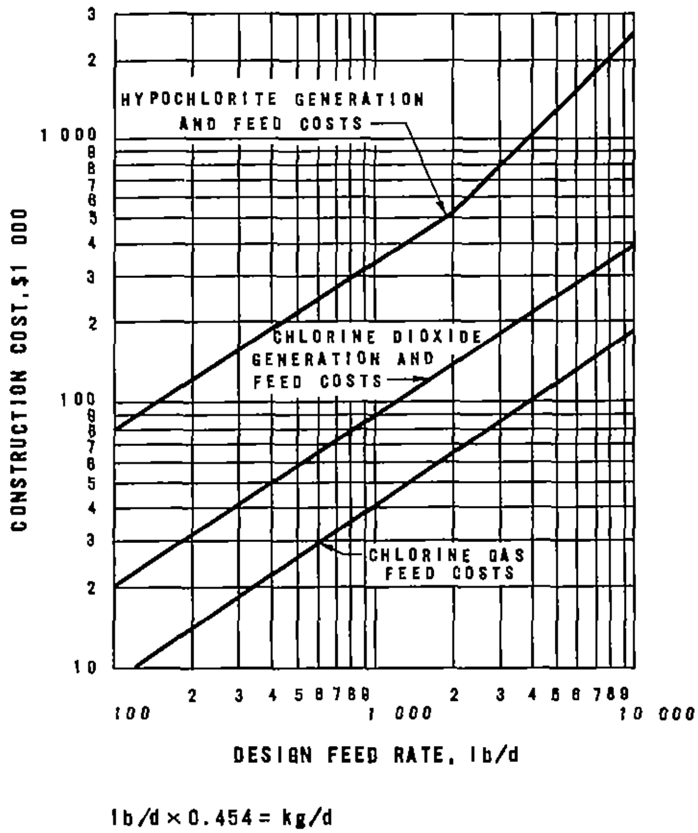


Figure 59. Chlorine disinfection cost curves, ENR 2000 [27].

EXAMPLE PROBLEM 7-1: ASSESSMENT OF STORAGE AND STORAGE/SEDIMENTATION COSTS AND COST EFFECTIVENESS

Given a frequency curve of storm rainfall, determine the costs, annual pollution reduction, and cost effectiveness for a storage and a storage/sedimentation facility. The storage facility is to be design to capture 95% of the total annual runoff volume. The storage/sedimentation facility is to be designed to capture 50% of the total annual runoff volume and treat those flows exceeding storage capacity by sedimentation.

Specified Conditions

1. Drainage area = 1000 acres.

2. Average runoff coefficient = 0.50

3. Total annual rainfall = 44 in.

4. Average suspended solids (SS) concentration in runoff = 400 mg/L

5. Construction costs (ENR 2000) for earthen-lined reservoirs = $0.25/gal of volume, for concrete sedimentation tanks, $1.00/gal of volume.

Assumptions

1. The storage volumes are based on a frequency plot of total storm rainfall, as shown in Figure 7-1.

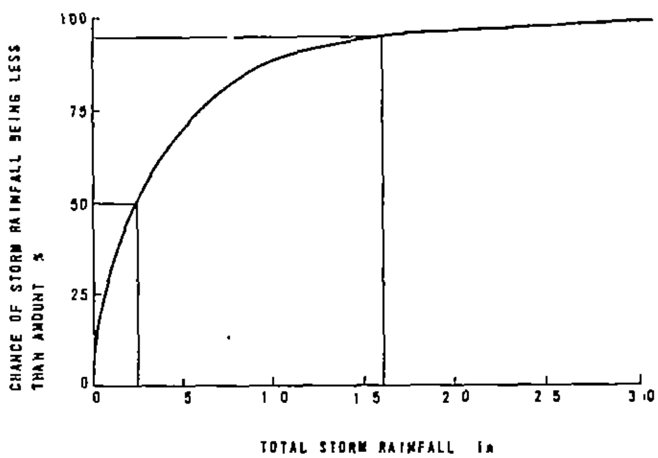


Figure 7-1. Percent chance of obtaining less than total storm rainfall amount.

2. It is assumed that runoff follows the same relationship of frequency as the rainfall curve.

3. Average flowrate to the storage/sedimentation facilities for flows exceeding storage capacity is based on the average of the maximum hourly rainfalls for each storm.

4. All flows totally contained in storage are to be released back to the interceptor and receive 85% removal at a dry-weather treatment facility.

Solution

1. Compute the volume and the construction cost for 95% storage of the annual runoff volume. From Figure 7-1, capture of storm rainfalls of less than 1.6 in. will result in a 95% capture of the annual rainfall volume.

   a. Determine design runoff amount using a 50% runoff coefficient.

      Design runoff amount = 1.6 in. x 0.50
                           = 0.80 in.

b. Determine storage volume required.

$$\text{Storage volume} = \frac{(0.80 \text{ in.} \times 1000 \text{ acres})\ (43\,560 \text{ ft}^2/\text{acre})}{12 \text{ in./ft}}$$
$$= 2.90 \times 10^6 \text{ ft}^3$$
or 21.7 Mgal

c. Compute the construction cost of the storage facility.

$$\text{Cost} = 21.7 \times 10^6 \text{ gal} \times \$0.25/\text{gal}$$
$$= \$5\,425\,000$$

d. Adjust ENR 2000 costs to current costs.

$$\text{ENR 2500 costs} = \$5\,425\,000 \times 1.25$$
$$= \$6\,780\,000$$

2. Determine the volume and the construction costs for 50% storage of the annual runoff volume for the storage/sedimentation facilities:

a. $$\text{Storage volume} = \frac{([0.24 \times 0.50] \times 1000)\ (43\,560)}{12}$$
$$= 435\,600 \text{ ft}^3$$
or 3.26 Mgal

b. Cost of storage facility = 3.26×10^6 gal x \$1.00/gal
= \$3 260 000

c. ENR 2500 cost = \$3 260 000 x 1.25
= \$4 075 000

3. Determine the total SS removed by the storage system capturing 95% of the annual runoff volume.

a. Compute annual runoff volume for a total annual rainfall of 44 in.

$$\text{Annual runoff volume} = \frac{([44 \times 0.50] \times 1000)\ (43\,560)}{12}$$
$$= 79.8 \times 10^6 \text{ ft}^3/\text{yr}$$
or 597 Mgal/yr

b. Compute annual SS load at 400 mg/L ≃ 400 ppm

$$\text{Annual load} = 597 \text{ Mgal/yr} \times 8.34 \text{ lb/gal} \times 400 \text{ ppm}$$
$$= 2 \times 10^6 \text{ lb/yr}$$

c. Compute the SS load contained in storage

$$\text{SS captured} = 2 \times 10^6 \text{ lb/yr} \times 0.95$$
$$= 1.9 \times 10^6 \text{ lb/yr}$$

d. Compute the SS removed by conventional treatment at a rate of 85%.

$$\text{SS removal} = 1.9 \times 10^6 \times 0.85$$
$$= 1.62 \times 10^6 \text{ lb/yr}$$

4. Determine the total SS removed by storage/sedimentation capturing 50% of the annual runoff volume and treating the remainder by sedimentation.

a. Compute the annual SS load contained in storage and treated at a conventional dry-weather facility achieving 85% removal.

$$\text{SS removal} = 2 \times 10^6 \text{ lb/yr} \times 0.50 \times 0.85$$
$$= 850\,000 \text{ lb/yr}$$

b. Determine the average flowrate for flows that exceed storage capacity using an average maximum hourly rainfall of 0.20 in./h.

$$\text{Runoff rate} = \frac{(0.20 \text{ in./h} \times 0.50)\ (1000 \text{ acres})\ (43\,560 \text{ ft}^2/\text{acre})\ (7.48 \text{ gal/ft}^3)\ (24 \text{ h/d})}{12 \text{ in./ft}}$$
$$= 65.2 \text{ Mgal/d}$$

c. Determine surface area of the storage/sedimentation basin at a 10 ft sidewater depth (swd).

$$\text{Area} = \frac{435\ 600\ ft^3}{10\ ft} = 43\ 560\ ft^2$$ (see 2.a.)

d. Compute average hydraulic loading rate.

$$\text{Hydraulic loading rate} = \frac{65.2 \times 10^6\ gal/d}{43\ 560\ ft^2} = 1500\ gal/ft^2 \cdot d$$

e. Determine the average SS removed by sedimentation at a hydraulic loading rate of 1500 $gal/ft^2 \cdot d$. Using Figure 38, SS removal = 30%.

$$\text{SS removed by sedimentation} = 2 \times 10^6\ lb/yr \times 0.50 \times 0.30 = 300\ 000\ lb/yr$$

f. The total SS removed by the storage/sedimentation facilities is 850 000 + 300 000 = 1.15×10^6 lb/yr.

5. Estimate the annual costs, including various land costs for storage and storage/sedimentation. Also determine the cost effectiveness for each type of storage.

a. Determine the gross land area requirements for storage, using a 10 ft swd and the typical section of an earthen embankment as shown in Figure 7-2.

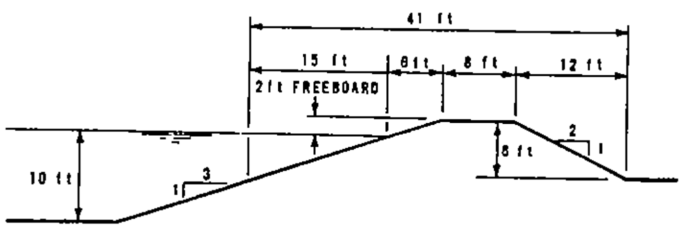


Figure 7-2. Typical earthen embankment detail.

$$\text{Effective water surface area} = \frac{2.9 \times 10^6\ ft^3}{10\ ft} = 290\ 000\ ft^2$$ (see 1.b.)

or 538.5 ft x 538.5 ft

$$\text{Gross area} = (538.5 + [2 \times 41])^2 = 385\ 000\ ft^2$$

or 8.84 acres

b. The area required for storage/sedimentation = $\frac{43\ 560\ ft^2}{43\ 560\ ft^2/acre}$ = 1.0 acre
No additional area is required because of the vertical concrete walls.

c. Estimate the land cost for storage at $10 000/acre.

Land cost = 8.84 acres x $10 000/acre
= $88 400

d. Compute the total construction and land cost for storage.

Cost = $6,780 000 + $88 400
= $6 868 400

e. Compute the amortized construction costs for storage using a 20 yr life at 7% interest.

Amortized construction cost = total cost x capital recovery factor (20,7)
= \$6 868 400 x 0.09439
= 648 000

f. The amortized construction costs for both storage and storage/sedimentation using land costs of \$10 000/acre, \$25 000/acre, and \$50 000/acre are summarized as follows:

Amortized construction costs, \$/yr			
	Land costs, \$/acre		
	10 000	25 000	50 000
Storage	648 000	661 000	682 000
Storage/sedimentation	386 000	387 000	389 000

g. Determine the cost effectiveness using amortized construction costs together with the total pounds of SS removed per year for the two types of storage at each land cost. The cost effectiveness for storage at \$10 000/acre $= \frac{\$648\ 000/yr}{1.62 \times 10^6\ lb/yr}$
$= \$0.40/lb$

Cost effectiveness values for all determinations are summarized as follows:

Cost/lb SS removed, \$/lb			
	Land costs, \$/acre		
	10 000	25 000	50 000
Storage	0.40	0.41	0.42
Storage/sedimentation	0.34	0.34	0.34

Comment

Although actual construction and land costs will vary from the values in this example, it can be seen that land costs affect storage costs and cost effectiveness to a greater degree than storage/sedimentation. A higher percentage of large total rainfall would require even larger storage facilities; however, if the majority of total rainfall volumes were small, total storage may approach the most economical and cost-effective solution.

EXAMPLE PROBLEM 7-2: DESIGN OF A SWIRL CONCENTRATOR/REGULATOR

Using the design curves developed from model studies [29], determine the design details for a swirl concentrator/flow regulator removing 90% settleable solids, and indicate the range of removals over the range of influent flows. Also, develop revised design dimensions using a weir height (H) equal to the inlet dimensions (D_1).

Specified Conditions

1. The design flow = 40 ft^3/s

2. The influent sewer size = 3 ft

Assumptions

1. The peak flow = 90 ft^3/s

Solution

1. Determine the standard design details (H_1/D_2 = 0.25) for the swirl concentrator/regulator.

   a. From Figure 7-3 (Figure 7 in reference [29]), determine the chamber diameter (D_2) for a design flow of 40 ft^3/s with a chamber inlet dimension of 3 ft (D_1).

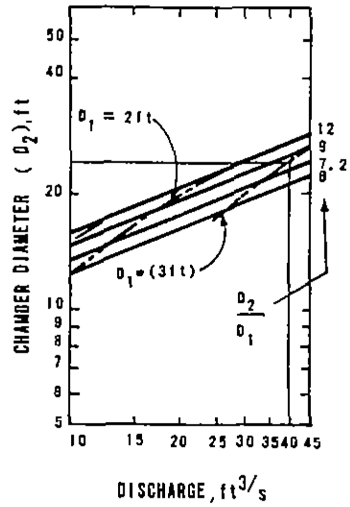


Figure 7-3. Swirl chamber diameters for 90% settleable solids recovery [29].

D_2 = 24 ft

   b. From Figure 7-4 (Figure 15 in reference [29]), check the settleable solids removal efficiency for a 24 ft diameter chamber.

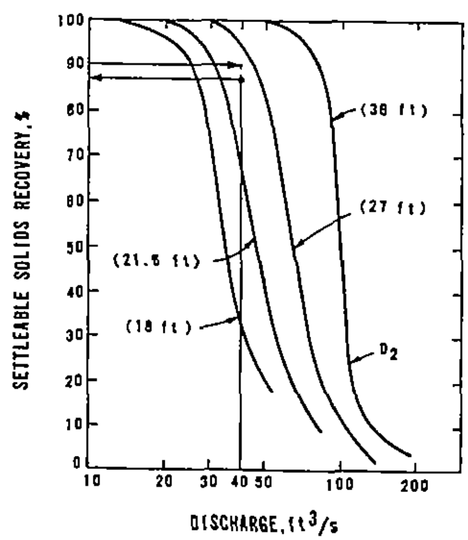


Figure 7-4. Settleable solids recovery for D_1 = 3 ft at H_1/D_2 = 0.25 [29].

Interpolating the recovery curve for D_2 = 24 ft, the swirl efficiency = 87%

c. Adjust the swirl chamber diameter to achieve 90% removal. From Figure 7-4, the D_2 dimension is interpolated from the curves at 90%.

Adjusted D_2 = 25 ft

d. Compute height of the swirl chamber (H_1) from relationship $H_1/D_2 = 0.25$.

$H_1 = 0.25 \times 25$ ft
$H_1 = 6.25$ ft

e. Determine the standard design details as shown in Figure 47, using the D_1, D_2, and H_1 values derived above:

$D_3 = 0.67 \times D_2 = 16.7$ ft
$D_4 = 0.56 \times D_2 = 13.9$ ft
$h_1 = 0.50 \times D_1 = 1.5$ ft
$h_2 = 0.33 \times D_1 = 1.0$ ft
$b_1 = D_2 \div 18 = 1.4$ ft
$R_1 = 0.39 \times D_2 = 9.7$ ft
$R_2 = 0.25 \times D_2 = 6.2$ ft
$R_3 = 0.104 \times D_2 = 2.6$ ft
$R_4 = 0.188 \times D_2 = 4.7$ ft
$R_5 = 0.61 \times D_2 = 15.3$ ft

f. Estimate the settleable solids removal over a range of expected flow of 10 to 90 ft^3/s using Figure 7-4

10 ft^3/s - removal = 100%
40 ft^3/s - removal = 90%
50 ft^3/s - removal = 75%
60 ft^3/s - removal = 47%
70 ft^3/s - removal = 28%
90 ft^3/s - removal = 12%

2. Determine the revised swirl chamber dimensions using a weir height (H_1) equal to the inlet diameter (D_1) of the standard design. The revised swirl concentrator will have the same settleable solids removal efficiency as the standard design. The geometry modification is made utilizing Figure 7-5 (Figure 10 in reference [29]).

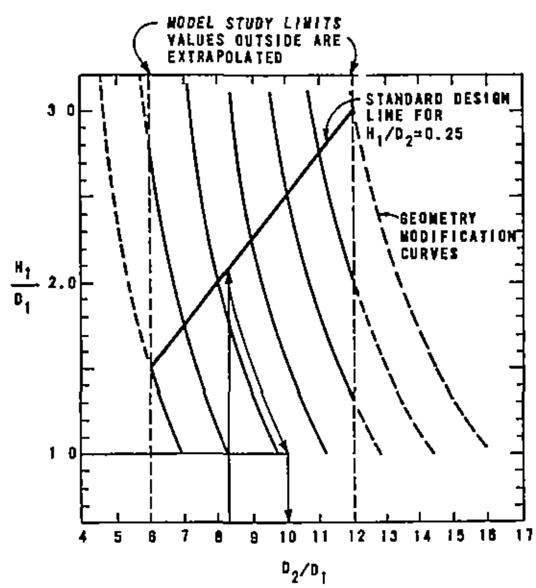


Figure 7-5. Swirl geometry modification curves [29].

a. Compute D_2/D_1 using the standard design values.

$D_2/D_1 = 25/3$
≈ 8.33

b. Enter Figure 7-5 at D_2/D_1 = 8.33 and move vertically to the standard design line. A revised D_2/D_1 value is obtained by moving along or parallel to the geometry modification curve to the specified H_1/D_1 value, in this case, H_1/D_1 = 1.0; and then down to the revised D_2/D_1 value of approximately = 10.0.

c. Compute the revised chamber diameter (D_2).

$$D_2 = 10.0 \times D_1$$
$$= 30 \text{ ft}$$

d. The other design dimensions are then recalculated using the new D_1, D_2, and H_1 values.

Comment

In detailing a swirl concentrator/regulator, the designer should choose a swirl inlet dimension approximately the same size as the influent sewer. However, where there is a choice of inlet sizes, the largest inlet size will result in the smallest, most economical structure. It is recommended that swirl designs also include an emergency overflow for flows that exceed peak design capacity. The swirl design curves developed from the model studies are limited by the fact that inlet dimensions of only 1 ft increments are provided for inlets 2 ft and larger; therefore, estimates of swirl size will have to be estimated or interpolated for odd sizes of inlets. The swirl design is also limited by the model study design limits for D_2/D_1 of 6 to 12.

EXAMPLE PROBLEM 7-3: ESTIMATION OF OPERATION AND MAINTENANCE COSTS FOR STORAGE FACILITIES

Develop a normalized operation and maintenance cost relationship such that average annual operation and maintenance costs may be estimated as a function of storage volume.

Specified Conditions

1. Storage volume, capital, and operation and maintenance costs for storage facilities are taken from Table 73.

2. Cost basis: ENR 2000.

Assumptions

1. Annual operation and maintenance costs are adjusted by the total storage capacity and the capital costs to obtain an equal basis of comparison, using the data for several sizes and types of storage facilities.

2. The resulting curves and equations represent an average normalization for any type and size of storage facility and are assumed to include labor, miscellaneous supply costs, and energy costs.

Solution

1. Determine the operation and maintenance cost factor (C_f) for the storage facilities presented in Table 73.

a. For Akron, Ohio, the C_f is evaluated by dividing the annual operation and maintenance cost by the storage capacity and the capital cost.

$$C_f = \frac{\$2\ 900}{(1.1 \text{ Mgal} \times \$455\ 700)}$$
$$= \$0.0058/\text{Mgal}\cdot\$ \text{ capital cost}$$

b. Operation and maintenance cost factors for the storage facilities are summarized as follows:

Annual operation and maintenance cost, \$	Storage volume, Mgal	Capital cost, \$	C_f \$/Mgal·\$
2 900	1.1	455 700	0.0058
51 100	3.9	1 774 000	0.0074
80 000	1.3	6 495 000	0.0095
97 600	1.2	9 488 000	0.0086
100 200	12.4	11 936 000	0.0007
2 700	2.8	744 000	0.0013
6 200	0.36	520 000	0.0331
3 340	0.20	883 000	0.0189
14 400	0.25	320 000	0.1800

c. The cost factors are plotted against storage volume, as shown in Figure 7-6, along with the best fit curve representing the average normalized conditions.

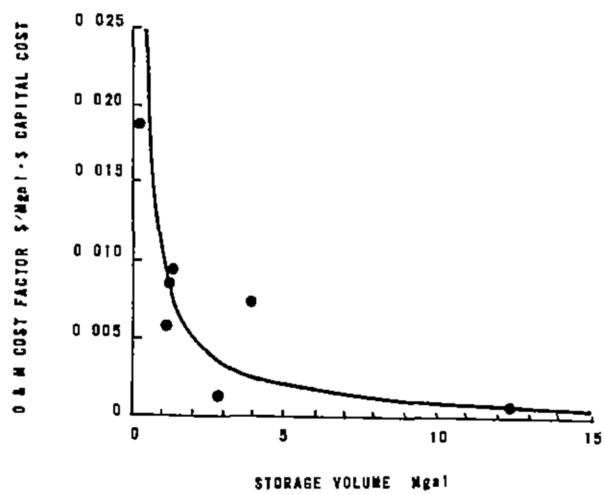


Figure 7-6. Operation and maintenance cost function for storage facilities.

d. The best fit curve has the equation:

$$C_f = 0.0105\ V^{-1.0476} \qquad (7\text{-}1)$$

where C_f = cost factor, \$/Mgal·capital cost
V = storage volume, Mgal

the correlation coefficient = 0.86

2. Develop a normalized operation and maintenance cost equation for storage facilities using the best fit curve equation from Figure 7-6.

Annual operation and maintenance costs are found by multiplying the cost factor at the required storage volume by the storage volume and the estimated capital cost:

$$\text{Operation and maintenance cost} = C_f \times V \times C_c \qquad (7\text{-}2)$$

$$= (0.0105\ V^{-1.0476})\ V \times C_c$$

$$= (0.0105\ V^{-0.0476})\ C_c$$

where V = storage volume, Mgal
C_c = capital cost, \$

3. Compare the results of the operation and maintenance cost equation with estimates obtained from the cost curves developed for stormwater facilities [27]. Capital costs for use in the equation are taken from the storage reservoir capital cost curve for concrete uncovered storage basins, Figure 34, to make an equal basis for comparing the operation and maintenance cost curves [27]. The comparison for storage facilities of 2, 5, 10, and 15 Mgal capacity is summarized as follows:

Storage volume, Mgal	Capital cost, $ (Figure 34)	Operation and maintenance cost, $/yr (Equation 7-2)	Operation and maintenance cost, $/yr[a] (cost curves [27])
2	500 000	5 100	5 880
5	900 000	8 750	7 950
10	1 300 000	12 200	11 300
15	1 700 000	15 700	13 600

a. Includes labor interpolated for 40 events per year at $10/h.

Comment

The operation and maintenance costs determined by Equation 7-2 provide a means and flexibility for estimating costs on a first-cut basis for both large and small storage facilities of simple or complex design and operation. Operation and maintenance costs based on the complexity of the design or process are controlled by the capital cost of the facility as well as the volume of storage. The operatión and maintenance values generated by the equation, using the capital cost values developed in reference [27], compare favorably with those taken from the curves.

EXAMPLE PROBLEM 7-4: STORAGE/TREATMENT OPTIMIZATION

Evaluate the cost of total treatment and total storage and determine the optimum storage/treatment combination for a given design rainfall at a level of treatment costing $30 000/Mgal·d.

Specified Conditions

1. Drainage area = 1000 acres.
2. Average runoff coefficient = 0.50
3. Capital cost for treatment = $30 000/Mgal·d
4. Capital cost for storage = $1.00/gal
5. Operation and maintenance costs for storage taken from Equation 7-2.
6. Operation and maintenance costs for treatment = 0 015 + 0.027 (treatment cost). Developed for reference [93].

Assumptions

1. Assume storage is to be dewatered in 24 hours.
2. The design rainfall rate = 1.2 in./h
3. The peak rainfall is assumed to be 1.5 x design rainfall.
4. The duration of rainfall equals runoff duration.

Solution

1. Determine the treatment capacity and cost to treat the total runoff. The treatment rate will be designed for the peak flow, without storage or flow attenuation.
   a. Peak rainfall = 1.5 x design rainfall
      = 1.5 x 1.2 in.
      = 1.8 in./h
   b. Determine the peak treatment rate (Q).

$$Q = \frac{(1.8 \text{ in./h} \times 0.50 \times 1000 \text{ acres})\ (43\ 560 \text{ ft}^2/\text{acre})\ (7.48 \text{ gal/ft}^3)\ (24 \text{ h/d})}{(12 \text{ in./ft})\ (1.0 \times 10^6 \text{ gal/Mgal})}$$
$$= 586 \text{ Mgal/d}$$

c. Compute the cost of treatment.

Cost = 586 Mgal/d x \$30 000/Mgal·d
= \$17.58 million

2. Determine the cost of storage assuming the stored volume is dewaterd in 24 hours through treatment costing \$30 000/Mgal·d. Using Equations 4-1a and 4-2a:

a. *Storage volume* = $0.02715\ KiAt_1 - (Q \div 24)t_2$: Eq.4-1a

$$= 0.02715 \times 0.50 \times 1.2 \times 1000 \times 1.0 - (Q - 24) \times 1.0$$
$$= 16.29 - (Q \div 24)$$
$$= 16.29 - (16.29 \div 24)$$
$$= 15.61 \text{ Mgal}$$

b. Cost of storage/treatment = $0.02715\ K_1At_1C_2 + Q\left[C_1 - \frac{C_2t_2}{24}\right]$: Eq. 4-2a

$$= 16.29 \times 1.0 + 16.29\left[0.03 - \frac{1.0 \times 1.0}{24}\right]$$
$$= 16.29 - 0.19$$
$$= \$16.10 \text{ million}$$

c. Evaluate the cost of storage and treatment individually for this situation.

Storage cost = 15.61 Mgal x \$1.0/gal = \$15.61 million
Treatment cost = 16.29 Mgal/d x \$0.03/gal·d = \$0.49 million

3. Determine the optimum storage/treatment combination using annual capital costs and total annual costs (including operation and maintenance).

a. Compute the storage volume required to reduce the peak treatment rate to the average design treatment rate, using the linear relationship shown in Figure 7-7.

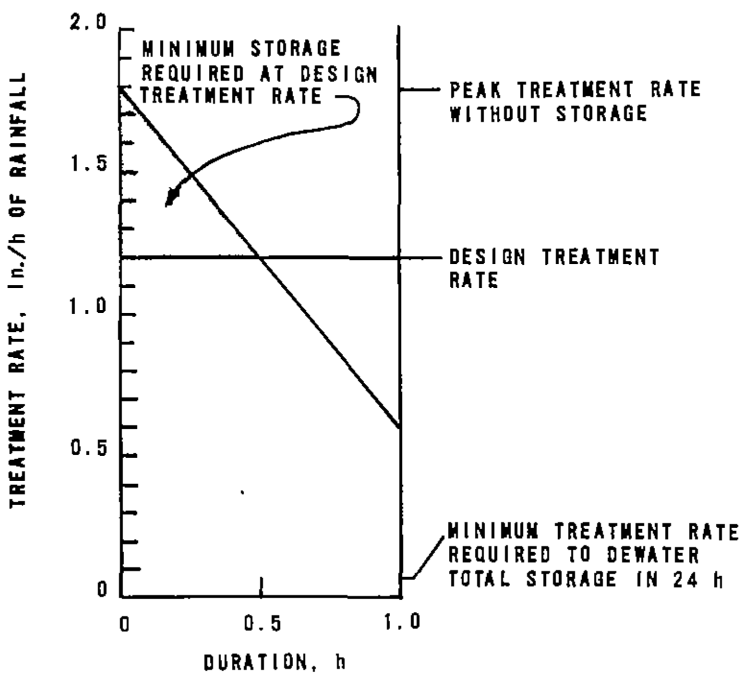


Figure 7-7. Relationship of treatment rate and storage volume for treatment rates greater than 0.6 in./h of rainfall.

The shaded area represents the storage volume required to provide the average design treatment rate of 1.2 in./h of rainfall. Area = 1/2 bh.

$$\text{Storage volume} = \frac{(0.5[1.8 \text{ in./h} - 1.2 \text{ in./h}] \times 0.5 \text{ h}) (0.50) (1000 \text{ acres}) (43\ 560 \text{ ft}^2\text{/acre}) (7.48 \text{ gal/ft}^3)}{(12 \text{ in./ft}) (1.0 \times 10^6 \text{ gal/Mgal})}$$

= 2.04 *Mgal*

b. Compute the treatment rate at 1.2 in./h of rainfall.

$$\text{Treatment rate} = \frac{(1.2\ \text{in./h} \times 0.50 \times 1000\ \text{acres})\ (43\ 560\ \text{ft}^2/\text{acre})\ (7.48\ \text{gal/ft}^3)\ (24\ \text{h/d})}{(12\ \text{in./ft})\ (1.0 \times 10^6\ \text{gal/Mgal})}$$

$$= 391\ \text{Mgal/d}$$

c. Determine the cost of storage and treatment at the design treatment rate.

Storage cost = 2.04 Mgal x \$1.0/gal = \$2.04 million
Treatment cost = 391 Mgal/d x \$0 03/gal·d = \$11.73 million

Total cost = 2.04 + 11.73 = \$13.77 million

d. Compute the storage/treatment costs for other treatment rates.

Note: At treatment rates of less than 0.6 in./h of rainfall, Equations 4-1a and 4-2a may be used. At treatment rates greater than 0.6 in./h of rainfall, the storage volume is computed from Figure 7-7 by multiplying the area of the triangle at the desired treatment rate by the appropriate conversion factors.

Costs of several storage/treatment combinations are as follows:

Rainfall, in./h	Treatment rate, Mgal/d	Storage volume, Mgal	Treatment cost, \$ million	Storage cost, \$ million	Total cost, \$ million
0.05	16 29	15.61	0.49	15 61	16.10
0.1	33	14.92	0.99	14.92	15.91
0.2	65	13.58	1.95	13.58	15.53
0.4	130	10 87	3.90	10.87	14.77
0.5	163	9.50	4.89	9.50	14.39
0.6	195	8.15	5.85	8.15	14.00
0.7	228	6.84	6.84	6.84	13.68
0.8	261	5.66	7.83	5.66	13.49
0.9	293	4.58	8.79	4.58	13.37
1.0	326	3.62	9.78	3.62	13.40
1.1	358	2.77	10.74	2.77	13.51
1.2	391	2.04	11.73	2.04	13.77
1.3	424	1.41	12.72	1.41	14 13
1.4	456	0 91	13.68	0.91	14.59
1.5	489	0 51	14.67	0.51	15.18
1.6	521	0.23	15.63	0.23	15.86
1 7	554	0.06	16.62	0.06	16.68
1.8	586	0	17.58	0	17.58

e. The total capital costs are coverted to amortized capital costs assuming a 20 year life at 7% interest. Compute the annual capital cost at a treatment rate of 16.29 Mgal/d.

Annual cost = \$16.10 million x 0.09349
= \$1.520 million/yr

f. Compute the annual operation and maintenance costs for each storage/treatment combination. The storage and treatment operation and maintenance costs at a treatment rate of 16.29 Mgal/d is computed below:

$$\text{Storage operation and maintenance} = 0.0105 \times 15.61^{-0.0476} \times 15.61 \qquad (7\text{-}2)$$
$$= \$0.144\ \text{million/yr}$$

$$\text{Treatment operation and maintenance} = 0.015 + (0.027 \times 0.49)$$
$$= \$0.028\ \text{million/yr}$$

g. Determine the total annual cost for each storage/treatment combination. The total annual cost for a treatment rate of 16.29 Mgal/d is determined below:

Total annual cost = 1.520 + 0.144 + 0.028
= \$1.692 million/yr